International Journal of Integrated Engineering, Vol. 7 No. 1 (2015) p. 12-20

The Determination Of Scaling Factor of Clay Properties On One-gravity Small Scale Physical Modeling.

Agus Sulaeman[1,*], Wan Nor Hanisah Wan Hamzah[2], Amirul Hafiz Ghazali[3]

[1]Faculty of Engineering Technology, Universiti Tun Hussein Onn Malaysia, 86400 Johor, MALAYSIA

Abstract: The observations and tests under small scale in 1-gravity condition is intended to obtain a comparative behavior of model and prototype of geotechnical case by imposing the scaling relations. Simulation to represent related structure, sub-soil and failure mechanism need to be prepared prior to do observations in this modelling. To obtain the new parameter for sub-soil simulation and inter-dependency with scaling relationship, the ten samples with different water content of prototype clay soil were consolidated in the triaxial CU test. After consolidation, each sample were given the arbitrarily initial mean stress $p_o = 1/3\ (\sigma_1 + \sigma_2 + \sigma_3)$ at the same time each corresponding void ratio were recorded. The data was plotted and numbered in the e Ln p' axises to adopt critical state line concept. Further shear stage in triaxial CU test were done to record the stress and strain of each ten samples. Among those of ten stress strain curves there were 3 similar curves (1, 6 and 8) observed when the deviatoric stress was normalized with its p_o, this showed similar behavior among them. The further observation revealed that void ratio in the clay soil no. 8 (e_p) corresponded with void ratio of the sample no. 1 (e_m), stress ratio N and critical state line parameter λ in the form of $e_m = e_p + \lambda$ Ln N. To support the expression of $e_m = e_p + \lambda$ Ln N, The "pile loading test " case was prepared in small scale and full scale modeling, e_m represented void ratio of clay in small scale and e_p represented void ratio of clay at original project location. Load settlement curves were obtained from both "pile loading test" in small and full scale simulation and the result showed closely good agreement.

Keywords: similarity, scaling factor, clay, small scale modeling, void ratio, critical state line

1. Introduction

In a small scale physical modelling, there are two opinions in relation with the use of scaling factor. One side only applies the geometry factor to reduce the size from prototype into small size of the model. This is intended and limited to model a simple geotechnical case problem [16]. The other side utilizes not only geometry, but also other scaling factors (stress, force, weight, time, velocity, void ratio etc.). Those scaling factors are used depending on the complexity of the case, and is intended to obtain the similarity behavior between model and prototype [11].

Reducing the size of the real object/area into small size (or utilizing geometry scaling factor) might be found in mapping terrain into map; however, simulating geotechnical case in small scale should consider another factors [17]. The followings are examples of the need of not only geometry scaling factor (n):

Stress analysis of Shallow Foundation

As shown in the Figure 1, the square shallow foundation (size B x B) under loading P.

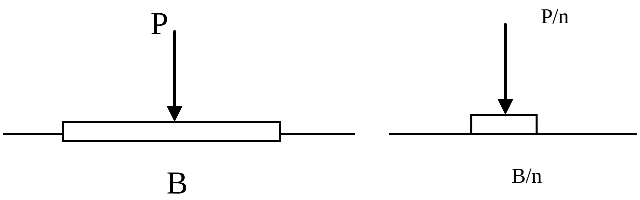

Figure 1: Prototype and Small scale basis model

There are 2 conditions frequently encountered in the research :

i) P and B reduced by n
The stress under the base of prototype footing is

$$\sigma = \frac{P}{B^2} \qquad (1)$$

stress under the base of scaled footing is

$$\sigma = \frac{P/n}{\left(\frac{B}{n}\right)x\left(\frac{B}{n}\right)} = \frac{P}{B^2}n \qquad (2)$$

*Corresponding author: agus@uthm.edu.my

penerbit.uthm.edu.my/ojs/index.php/ijie

The stress at both location is different and this leads to inconsistency.

ii) P unchanged, B reduced by n

$$\sigma = \frac{P}{(\frac{B}{n})x(\frac{B}{n})} = \frac{P}{B^2}n^2 \qquad (3)$$

The stress at both locations is significantly different. In order to obtain the similarity behaviour between model and prototype, the stress at both models should be made similar; therefore, in this footing case, other than n, stress scaling factor should be introduced.

Safety Factor of Slope Stability

The formula to determine safety factor of a slope SF is :

$$SF = \frac{C_u R^2 \theta}{Wd} \qquad (4)$$

where C_u is undrained shear strength

The failure line in full scale as shown in Figure 2 frequently encountered in the same pattern at the small scale modeling. Following the same SF formula with prototype, the calculated SF in small scale will deviate.

To obtain the same SF to support similar behaviour, other than n, there are other scaling factors to consider i.e: C_u and weight of soil, W. Both parameters are inherently occupy the soil characteristic; therefore, the modification to soil properties is imperative.

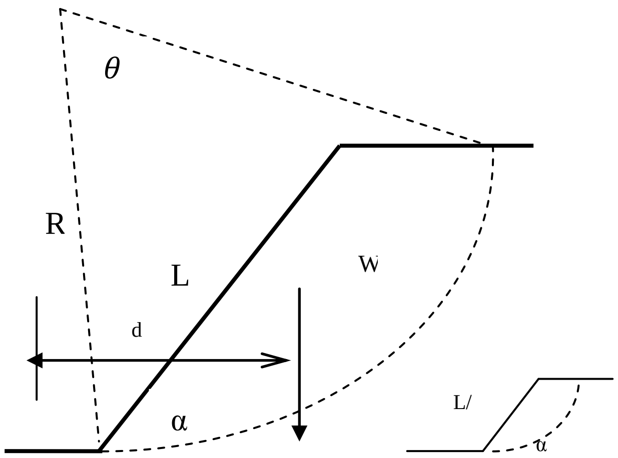

Figure 2 : Slope stability case

Retaining structures case

In order to obtain the similarity performance of model with prototype, stress at homologous point should be similar. For example, stress at point A in both prototype and model as shown in the Figure 3. The formula to calculate active pressure at point A in both models is :

$$\sigma_a = \gamma H \frac{1-\sin\emptyset}{1+\sin\emptyset} \qquad (5)$$

The result will be different when the scaling factor for γ and $\emptyset$ are not taken into account.

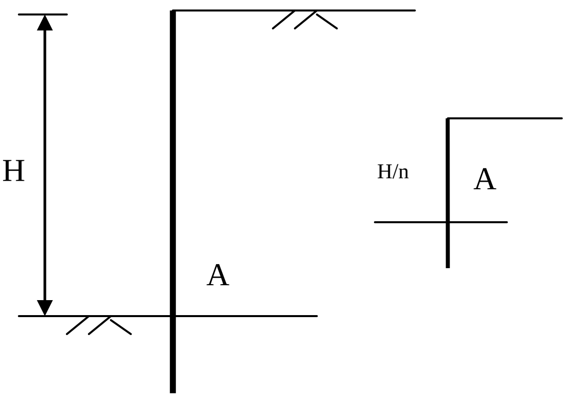

Figure 3 : Earth retaining structure case

From the above examples, the stress scaling factor is dominant. In 1930's the centrifugal devices was created to accommodate this factor [12]. Due to capital extensive in setting up this system, many attempts have been made to come up with a cheaper and effective system. The followings are some of them:

NGI, 1981

Norwegian Geotechnical Institute in collaboration with Conoco Phillips have conducted scaled modeling using modified triaxial device to obtain the behaviour of single pile under cyclic and lateral load in an offshore structure.

The model pile is inserted into triaxial chamber as replacement to soil sample, then confining pressure as simulation of overburden pressure in real site is applied subsequently. The performance of pile is monitored during application of deviatoric stress[10].

Increased hydraulic gradient

The increased gradient method scales the vertical stress distribution by imposing a downward flow with a large, positive pressure gradient in the pore fluid in saturated soils used in the model. For a soil that is subjected to a vertical pore fluid pressure gradient i , the effective stress the soil will be :

$$\sigma' = \sigma - \gamma_w h(1-i) \qquad (6)$$

Where σ' is the effective stress in the model
σ is the total stress in the model
$\gamma_w h$ is the hydrostatic head of the fluid

in the model
i is pore fluid pressure gradient in the model, defined positive in the downward flow and negative in upward flow.

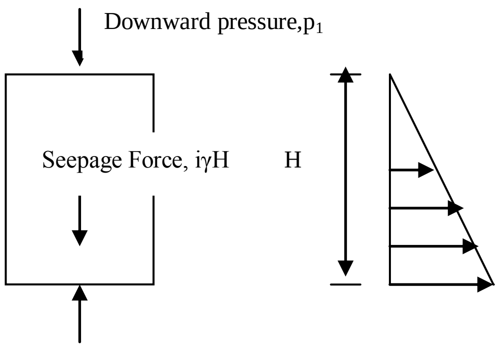

Figure 4 : The concept of increased hydraulic gradient

γ' = submerged unit weight of soil
i = hydraulic gradient
γ_p = unit weight of prototype soil

$$N' = \frac{\gamma_e}{\gamma_p} = \frac{(i\gamma_w + \gamma')}{\gamma_p} \quad (7)$$

Where $\gamma_p = \gamma'$ so $N' = \dfrac{i\gamma_w + \gamma'}{\gamma'}$

Whereas $i\gamma_w >>> \gamma'$ and $\gamma_w \cong \gamma'$
so $N' = i$

N' = hydraulic gradient scaling factor.

According to Zelikson [18] for proper modeling, the product of geometric scale ratio n and the stress gradient ratio I must be equal to unity. Then, the displacement ratio between the model and prototype will be equal to the geometric scale ratio n. This method was intended to overcome the problems associated with 1-g model tests that is the stress at all homologous point of the model is equal to the stress induced by gravity in the actual prototype. By imposing a powerful downward gradient of pore fluid, but limited to certain type of soil and situations.

Altae and Fellenius, 1994

In 1994, Altae and Fellenius reported that some examples of small scale test on bearing capacity of footings which have been published by many researchers are unreliable due to the mistake in using similar void ratio (density) of model soil and prototype soil. Then, they presented the new approach 1-g modeling in non-cohesive soil resulted in the use of void ratio of soil model should be different from prototype soil:

$$e_m = e_p + \lambda \ln N \quad (8)$$

e_m = void ratio of the soil model
e_p = void ratio of the soil at prototype
λ = critical state line (CSL) slope
N = stress scaling factor

Fellenius and Altae (1994) reported that Roscoe et al. (1968) developed the Casagrande concept of critical void ratio and critical density into defining a state at which the soil continues to deform at constant stress and constant void ratio, calling this state the " critical state ". This concept was based on the results of extensive laboratory testing of remolded clays. The approach was later found valid also for non cohesive soils as mentioned by Atkinson and Bransby [3].

Frustrum Confining Vessel (FCV) by Horvath and Stolle (1996)

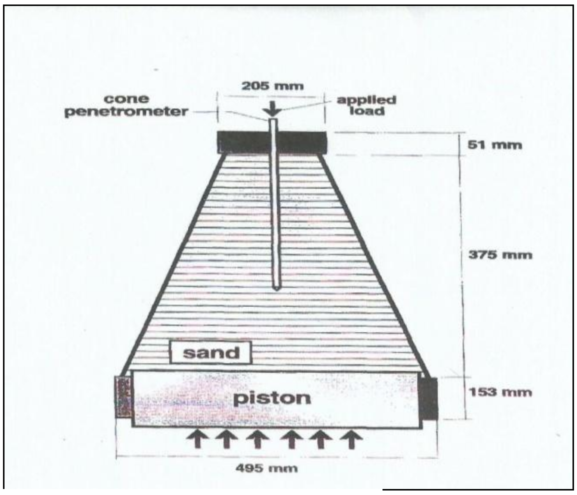

Figure 5 : Frustum confining pressure

A cone-shaped confining vessel for testing small scale model piles was developed by Horvath and Stolle [8] which controls the confining stress within the model soil mass by applying a vertical stress at the bottom of the soil mass. The cone-shaped confining vessel is technically a frustum, the part of a cone left after the top has been cut off parallel to the base.

Owing to the conical shape of the frustum confining vessel (FCV), the vertical stress in the soil at the top is zero (which corresponds to the ground surface) and it increases with depth to the value of stress applied by the bottom piston. The lateral stresses within the model soil mass, which are a function of the vertical stress and mechanical properties of the soil (friction angle), also increase with depth. Thus, a model pile may be tested under stress conditions that compare more closely with stresses occurring in full scale foundations. A hydraulic jack presses against the bottom of the piston to achieve the desired stress levels within the confined soil.

Scaling factors is used in scaled modeling depending on a case problem to be observed,
Table 1 shows the scaling factors normally implemented in 1-gravity and enhanced gravity environments. It has been mentioned by Fellenius [1,7] that the formula of $e_m = e_p + \lambda \ln(N)$ is a void ratio in the model to simulate sand. However, the usage of this formula to simulate cohesive soil is yet to be revealed.

To determine the other scaling factor for similarity requirements, for example scaling factor of "time" in pile loading test simulation, we need theoretical and mathematical approach which suits to this case problem. The object in nature which can represent the pile motion during pile loading test can be simulated by equation of motion of the object as mentioned by Sedran [13]:

In full scale (prototype) :

$$M_p \ddot{A}_p + C_p \dot{A}_p + K_p A_p = F_p (t_p) \quad (9)$$

In model (reduced scale) :

$$M_m \ddot{A}_m + C_m \dot{A}_m + K_m A_m = F_m (t_m) \quad (10)$$

In general, for any given similarity analysis the following scaling factors apply to the equation of motion.

Mass : $\lambda_m = M_m / M_p$ (11)

Damping : $\lambda_c = C_m / C_p$ (12)

Stiffness : $\lambda_k = K_m / K_p$ (13)

Force : $\lambda_f = F_m / F_p$ (14)

Displacement : $\lambda_L = L_m / L_p$ (15)

Velocity : $\lambda_v = V_m / V_p$ 16)

Acceleration : $\lambda_a = A_m / A_p$ (17)

Time : $\lambda_t = t_m / t_p$ (18)

Substitution (11) to (18) into (10) :

$M_m = \lambda_m M_p$; $C_m = \lambda_c C_p$; $K_m = \lambda_k K_p$;
$F_m = \lambda_f F_p$; $L_m = \lambda_L L_p$
$V_m = \lambda_v V_p$; $A_m = \lambda_a A_p$; $t_m = \lambda_t t_p$

$$\{\lambda_m \lambda_a\} M_p \ddot{A}_p + \{\lambda_c \lambda_v\} C_p \dot{A}_p + \{\lambda_k \lambda_L\} K_p A_p = \{\lambda_F\} F_p (t_p) \quad (19)$$

However : $\lambda_V = \lambda_L / \lambda_t$; $\lambda_a = \lambda_L / \lambda_t^2$;
$\lambda_m = \lambda\rho \,.\, \lambda_{vol} = \lambda\rho \,.\lambda_L^3$

Provided that we enforce the condition $\lambda\rho = 1$ (assuming density of the model similar to that of prototype) we can express $\lambda_m = 1.\ \lambda_L^3$ or
$\lambda_m = \lambda_L^3$,

Then in equation (17)

$$\{\lambda_{vol}^3 \lambda_L/\lambda_t^2\} M_p \ddot{A}_p + \{\lambda_c \lambda_L/\lambda_t\} C_p \dot{A}_p + \{\lambda_k \lambda_L\} K_p A_p = \{\lambda_F\} F_p(t_p) \quad (20)$$

Dividing by $\{ \lambda_F \}$:

$$\{ \lambda_L^4/\lambda_t^2 .1/\lambda_F \} M_p \ddot{A}_p + \{\lambda_c \lambda_L/\lambda_t \lambda_F\} C_p \dot{A}_p + \{\lambda_k \lambda_L/\lambda_F\} K_p A_p = F_p (t_p) \quad (21)$$

For similarity to be fulfilled then the following conditions should be satisfied :

$$\{ \lambda_L^4/\lambda_t^2 .\ 1/\lambda_F \} = 1 \quad (22)$$

$$\{ \lambda_c \lambda_L/\lambda_t \lambda_F \} = 1 \quad (23)$$

$$\{ \lambda_k \lambda_L / \lambda_F \} = 1 \quad (24)$$

If model testing is done by a 1 – g environment, the scaling factor for acceleration = 1

$\lambda_a = A_m / A_p = 1$; $\lambda_a = \lambda_L / \lambda_t^2 = 1$; $\lambda_t^2 = \lambda_L$

$$\lambda_t = (\lambda_L)^{0.5} \quad (25)$$

Hence,
$t_m / t_p = (L_m / L_p)^{0.5}$; $L_m / L_p = n$
$t_m / t_p = (n)^{0.5}$

Table 1: Scaling relations of the physical modeling approach 1-g and centrifuge environments [9,10].

	Full scale prototype	Model
Linear dimension	1	n
Area	1	n^2
Stress	1	N in 1-g
	1	1 in centrifuge
Strain	1	1
Displacement	1	n
Force	1	Nn^2 in 1-g
		n^2 in centrifuge
Void ratio, sand	e_p	$e_m = e_p + \lambda \ln(N)$
time	t_p	depends on case problem

Where :
n = geometric scale ratio
N= stress scale ratio
e_m = void ratio model
e_p = void ratio prototype
g = gravity

To determine the specific scaling relations of other case, the approach to manipulate the corresponding parameters needs to be analyzed.

Clearly mentioned earlier that the non-centrifuge system were attempting to fulfill similarity requirements, among those of the systems, the utilization of critical state concept introduced by Altae and Fellenius was selected to simulate pile loading test (PLT) case.

2. Testing Program

The works consists of laboratory and field loading test works.

Laboratory Program

The following tests were done to the clay sample which was taken from UTHM Recess lab area at 2-3 m depth.

i. Determination of modeled soil properties

Prior to do tests to determine the modeled soil, the engineering properties and critical state line (CSL) of clay soil at same location was previously measured. The ten samples were taken from original clay soil of Recess UTHM field lab area. Each sample was consolidated in triaxial until certain mean stress $p_o = \frac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3)$, and it's void ratio was calculated. Then each sample was sheared un-drained. The data from this test was plotted in e vs Ln p and deviatoric stress vs strain to be analyzed to determine which curves coincide.

Once the analysis and calculation of soil model determined ($e_m = 1.95$, see Table 3), the original clay soil was modified. The modification was made by adding-up water into soil and mixed it up. The mix proportion was in such a way in order to produce void ratio e_m. Then the void ratio e_m in the box was maintained unchanged to prevent from extreme evaporation, the filled water tank was connected to this box.

ii. Set-up small scale model box

Small scale model box completed with necessary instrumentations was designed and then erected. The schematic illustration and as built modelling box device were shown in the Figure 6. This instrumented box was prepared and intended to set-up small scale physical model device.

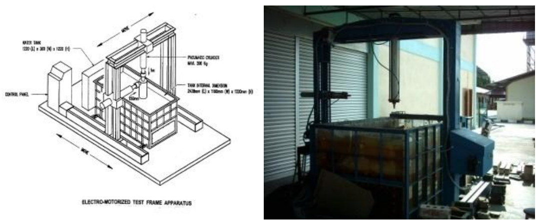

Figure 6 : (a)Schematic diagram and (b) as built of Small scale physical modeling box

iii. Driving Simulation

In order to prove that there was a relationship of scaling laws between model and prototype, pile loading test case was selected. To do so, the necessary aspects of modeling in small scale 1-g modeling should be prepared i.e.: modeled soil (e_m), modeled pile, pile driving and loading mechanism.

To simulate the reinforced concrete pile of the size 15 cm in diameter and length of 6 m, the aspect of scaling relation n = 10 was applied resulted in pile model having 1.5 cm width, length 60 cm and made from concrete mortar to represent similar roughness with real reinforced concrete pile

Void ratio of original clay soil e_p should be modified to fulfill similarity conditions, void ratio at the model box e_m should be reached as to replace original void ratio e_p.

In general, pile is driven by Pile Driver. Certain hammer weight is dropped to reach a desired pile set as shown in Figure 7. To simulate this, modeled pile was driven gradually by modelled hammer (actual hammer weight divided by scaling factor $n \times n \times n = n^3$) to reach full length embedded. In this stage, model of pile hammer was 1 kg to satisfy actual hammer weight of 1 ton.

The final pile set both in full and small scale tests were impossible to reach due to very soft clay condition before and after original clay soil was modified to reach void ratio, e_m.

Figure 7: Pile being driven at research location

iv. Pile Loading Test (PLT) Simulation

Once the modeled pile driven, the arrangement of instrumentations were then set up to follow the similar full scale loading test of PLT as shown in the Figure 8. Loading mechanism of pile loading test of Slow Maintain (SM) was mentioned on the ASTM standard D 4410[2]. This mechanism was then applied in small scale basis. Slow maintained loading test is normally performed on the measurement of load cell and displacement transducer in every 15 minutes.

The rate of loading when pile moving down during loading test was controlled in such a way that it follows the scaling factor of time, $t_p(n)^{0.5}$
or $t_m = 15(1/10)^{0.5} = 5$ minutes. (see Table 1). Other than that, the failure was based on one of the following condition :

1. 10 % of pile width achieved (1.5 mm)
2. No further resistance recorded in load cell
3. Displacement recorded in the data logger were detected high.

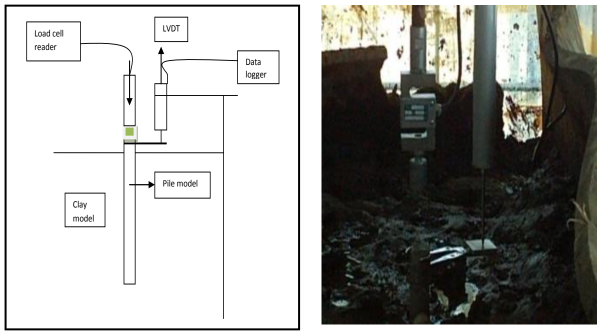

Figure 8 : a) Schematic PLT on small scale basis
b) Scaled pile loading test was undertaken

According to the normal practice of Slow Maintain (SM) test, 1-2 days were needed to perform this test. In this scaled basis, in accordance with the other requirement of failure state the test was slightly faster than time scale requirements.

Field Test Program

The field test was intended to measure the real capacity of the piles. Two square RC pile, 150 x 150 mm, 6 m length was driven to field lab area. Instead of using 1 pile, 2 piles were implemented to obtain the average and acceptable value to ensure the robustness of the result.

i. Driving the Piles

The two piles were driven based on the normal practice of driving until all pile length inserted.
Waiting period of 30 days before commencement of full scale loading test was implemented to allow dissipation of pore water pressure in the vicinity of the pile. This was done to obtain true capacity of pile due to the conditions of soft clay and high ground water level at field lab Recess area.

ii. Pile Loading Test

The concept of kentledge system was adopted to do pile loading test instead of using other method, since the capacity of this pile could be predicted based on the available site investigation data of the area. The PLT was done successfully as shown in Figure 9.

Figure 9: Full scale pile loading test was underway

3. Test Results, Analysis and Validation

The followings are data results, analysis and verification of scaling factor of clay soil:

Engineering properties

The engineering properties as shown in the Table 2 demonstrates the very soft clay and high plasticity.

Table 2 ; Engineering properties of Recess clay

Depth (m)	Class.	Atterberg Limit	Oedometer test	Triaxial CU
2.0 – 3	CH	MC=57 LL=68 PL=27 PI=41	C_v = 0.9 m^2/year C_c = 0.51	C= 7kPa Φ = 8

Meanwhile, the Critical State Line (CSL) value was calculated λ= 0.191 [14,15] had been investigated earlier by author, the result of CSL was shown in Figure 10.

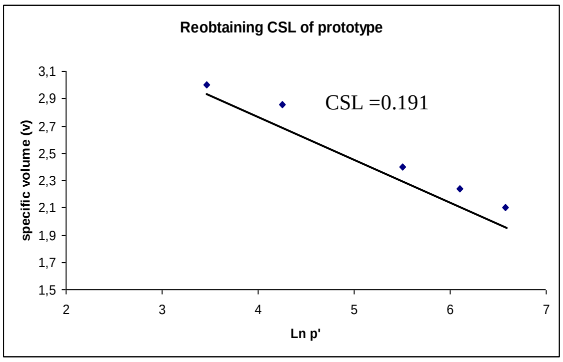

Figure 10 : CSL of clay sample of project location

To verify the correctness of CSL value [3], let refer to the expression of

$$\lambda = \frac{Cc}{2.303} \tag{26}$$

Hence, Cc = 0.191 x 2.303 = 0.44. This number is not too far with actual site condition of soft clay (Cc in the range of 0.40 to 0.55). It has been proven that the similarity of model and prototype is governed by the condition in e vs Ln p graph, similar behavior would occur when the two data is connected in one line parallel with CSL of original soil [4,5].

Lab work

Lab work result to obtain similarity consist of scattered 10 data. To comply with similarity, it is imperative to investigate which one of these scattered data is parallel to CSL. Based on the consolidated samples to its p_o and calculated each void ratio as shown in Table 3, the e vs Ln p_o is tabulated.

Table 3 : Initial mean stress and its void ratio

No	p_o	e
1	30	1.95
2	40	1.85
3	50	1.74
6	60	1.82
4	70	1.75
5	80	1.70
7	90	1.69
9	110	1.75
8	150	1.64
10	130	1.73

Figure 11 shows a result of plotted ten samples consolidated in triaxial and the Critical State Line of this clay soil. According to Fellenius [1,7], the similarity behavior of two samples would be found when σ-ε curve was similar or the deviatoric stress was normalized by initial mean stress p_o also coincide. To follow this, all σ-ε of ten data were observed and the results was negative, instead when the curves were normalized by it's p_o, it showed 3 curves of no.1 , 6 and 8 coincide.

Further observation to this three data, it reveals that $e_m = e_p + \lambda \text{ Ln } N$ for sand is also applicable for clay soil. By the guidance of the Figure 11 , the following is the calculation of it. Let void ratio of soil model represented by e_1 and void ratio of original clay prototype was e_8 or e_6 depending on how much N is planned.

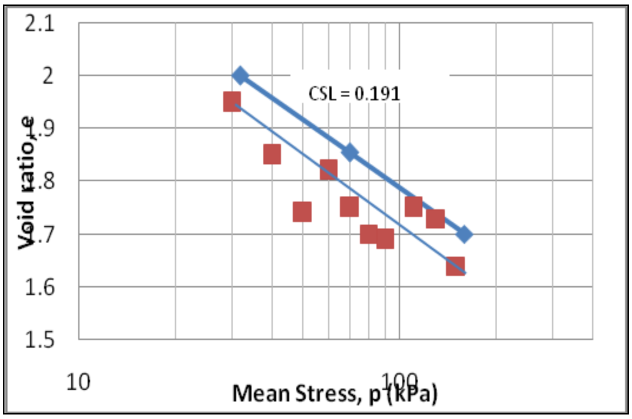

Figure 11 : Scattered mean stress vs void ratio

a) Sample 1 and 8

$e_1 = 1.95$; $e_8 = 1.64$; $\lambda = 0.191$

$e_m = e_p + \lambda \text{ Ln } N$; $1.95 = 1.64 + 0.191 \text{ Ln } N$

Hence, N = 5

For verification, $p_{o8} = 150$ kPa and $p_{o1} = 30$ kPa , N also ratio of p_{o8} to p_{o1}

b) Sample 1 and 6

$e_1 = 1.95$; $e_6 = 1.82$; $\lambda = 0.191$

$1.95 = 1.82 + 0.191 \text{ Ln } N$

Hence, N = 2 this is in accordance with ratio of $p_{o6} = 60$ kPa and $p_{o1} = 30$.

It is also noted that when sample 1, 6 and 8 are connected, it produces a line which is parallel to CSL. This is in conformity to the concept of similarity. Shown in the Figure 12, the three normalized curves is identical as to compare with other curve in Figure 13.

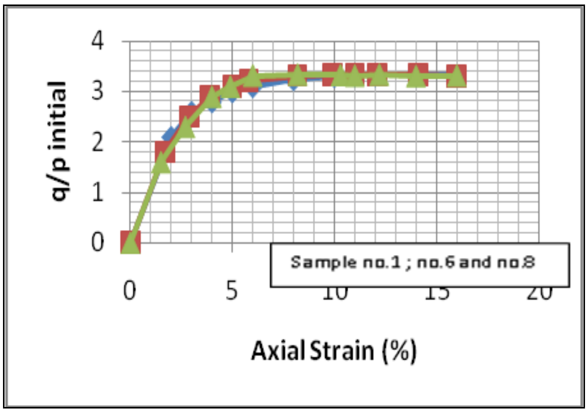

Figure 12: Normalized deviatoric stress to initial mean stress of sample 1,6 and 8

Thus, it can be concluded that stress scaling factor N resulted from modification of original soil into e_m in e – Ln p environment can be used to simulate stress ratio in 1-g environment.

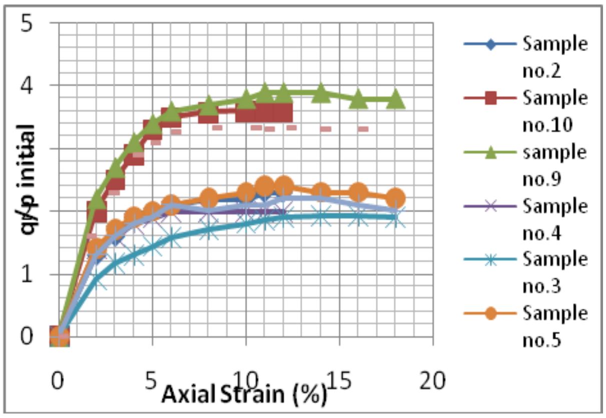

Figure 13: Normalized deviatoric stress to initial mean stress of all 10 samples

In enhanced gravity/centrifuge test, increased gravity is released to reach desired stress. However, stress level in 1-g model can be reached by modification an original prototype soil.

Small Scale Model

In order to ensure the accuracy of N, small scale and full scale of geotechnical case should be performed. In this research, the pile loading test of reinforced concrete pile was selected as mentioned earlier. In the small scale basis, the data which was taken from instrumentations was the raw data and should be converted by scaling factors as shown in the Table 4.

Table 4: Raw and converted data from small scale Instrumentation

Displacement transducer	load cell reading	Converted transducer	Converted load
mm	kg	mm	kN
21,0231	0,00	0,000	0,000
21,0181	0,40	0,005	2,000
20,9983	0,90	0,025	4,500
20,9486	1,30	0,075	6,500
20,8939	1,60	0,129	8,000
20,8442	2,10	0,179	10,500
20,8243	2,40	0,199	12,000
20,8094	2,80	0,214	14,000
20,7647	3,20	0,258	16,000
20,6456	3,40	0,377	17,000
20,1683	3,94	0,855	19,700
19,4476	4,10	1,576	20,500
16,1624	4,20	4,861	21,000
10,5215	3,80	10,502	19,000

Using scaling factor for geometry n = 10, and N = 5 as a result of soil modification, Load F become $Nxn^2 = 5 \times 10^2 = 500$. The converted 2 columns in Table 3 was the value from conversion by 10 and 500 for converted transducer and converted load respectively. It was noted that ultimate capacity of the pile was 21 kN as can be measured from converted L-S curve shown in Figure 14.

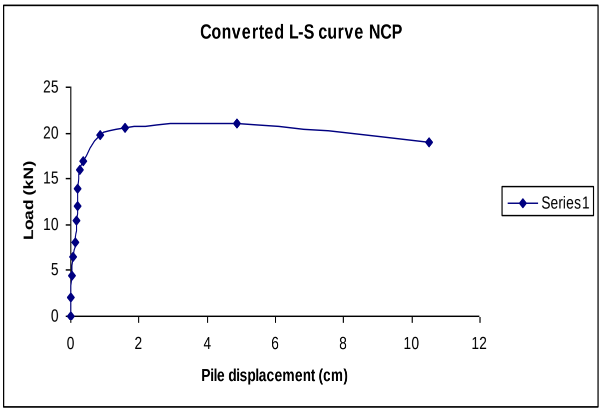

Figure 14 : Converted L-S curve

Full scale

In full scale pile loading test, all the readings was not necessarily converted. Due to the fact that the result of two piles tests were almost similar, only one data was revealed. Shown in the Figure 15 the ultimate capacity was 22 kN. Although the ultimate capacity from small scale and full scale almost similar, there is a difference in the onset of failure. The first is reaching ultimate at 1.5 cm displacement whereas the latter failed at 2.2 cm. The slight difference of the curves shown in the Figure 14 and 15 is possibly due to other scaling factor which is not taken into account i.e.: friction/ roughness and stress history of soil. It is not well established to scale down the roughness, the roughness measurement needs special equipment as well as to produce scaled roughness of concrete surface. To obtain stress history similar between model and prototype is also another difficulty. It might be concluded that many scaling factors to be considered is likely to be more accurate.

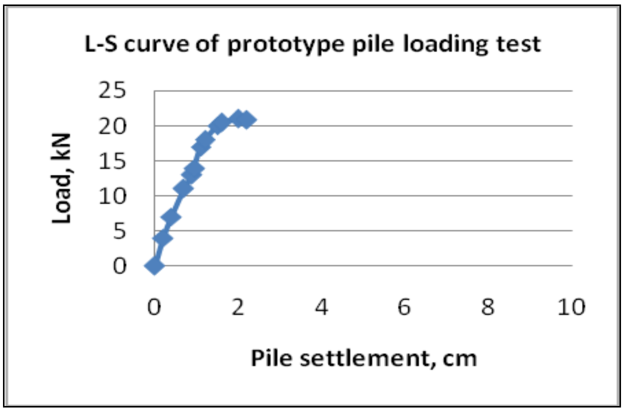

Figure 15 : L-S curve from full scale loading test

Validation

To compliment the scaled normal gravity modeling, full scale loading test should be done.
In order to verify the validity of this model, let analytical calculation using laboratory data be carried out to the reinforced concrete pile. There are some formula to compute pile ultimate capacity Q_u in clay soil [6], which consist of point bearing Q_p and Friction capacity Q_f .

Data of soil :
$C = 7.01$ kPa ; $\phi = 8.35^o$; $\gamma = 15.69$ kN/m^3 ;
PI = 41 ; GWL at 1m beneath ground surface

Pile data :
Length effective = 5.5 cm ; size = 15 x 15 cm.

Point bearing capacity

From Tomlinson (2001), Nc* = 9
$Q_p = Ap \times c \times N_c^* = 0.15 x 0.15 x 7.01 x 9 = 1.44$ kN

Friction Capacity

1) α method
From table to find α ; α = 1 and considering tension crack until 1.5 D = 22.5 cm. L become 5.5 m – 0.225 m = 5.275 m
$Q_f = L\, \alpha\, C_u\, p = 5.275 \times 7.01 \times 4 \times 0.15$
= 22.19 kN
$Q_u = Q_p + Q_f = 1.44 + 22.19 = 23.63$ kN.

2) Karlsrud method, consider PI and in situ effective stress.

$\sigma'_{vo} = 5.5\ (15.69\text{-}10) = 30.8$ kPa

$\frac{Su}{\sigma'_{vo}}$ = 7.01 /30.8 = 0.32

$\alpha = 0.32\ (PI - 10)^{0.3} = 0.83$
$Q_f = L\ \alpha\ C_u\ p$ = 5.275x0.83x7.01x4x0.15
=18.39 kN
$Q_u = Q_p + Q_f$ = 19.83 kN

The result of Q_u measured from PLT and small scale is around 21 kN. Although slightly deviated, the amount of different is not significantly big and this result is encouraging.

4. Conclusion

1. Simulation of geotechnical case in normal gravity to model geotechnical case problem need special attention to scaling factors.
2. The expression of $e_m = e_p + \lambda \text{ Ln } N$ can be applicable also for clay soil to modify original soil into model soil.
3. The requirement of parallel with CSL means that the small scale model test should be performed in soil that is looser than prototype soil. This imposes boundaries on the scaling relations because; First, a model test cannot be performed in a soil looser than critical void ratio. Second, a model test must not be performed in a soil denser than prototype soil. Clay soil with too high of water content (high void ratio) tends to be more in liquid phase.
4. Complete scaling factors would result in good accuracy, otherwise, less accuracy will be obtained.

Recommendations

1. This tests was basically done in triaxial CU, the drained tests condition is recommended.
2. Test to other type of clay soils is also recommended.

Acknowledgement

This research was financially supported by Recess UTHM, the writers are grateful to their supports and encouragements.

References

[1] Altae, A and Fellenius, B.H (1994), "Physical Modelling in Sand " Canadian Geotechnical Journal.
[2] ASTM D1143-81 (Reapproved 1994) Standard Test Method for Piles Under Static Axial Compressive Load.American Society of Testing and Material (ASTM), USA, pp. 768-778.
[3] Atkinson Bransby . (1978), "An introductionTo critical state soil mechanics " Mc Graw hill UK.
[4] Been, K. and Jefferies, M.G., 1985. A State parameter for Sands, Geotechnique, 35: 99-112.
[5] Been, K., Jefferies, M.G. and Hackey, J.,1991, The critical state of sands, Geotechnique, 41: 365-381.
[6] Doherty, P and Gavin, K (2011), The Shaft Capacity of Displacement Piles in Clay :A State of the Art Review, Journal of Geotechnics and Geological Engineering, Springer.
[7] Fellenius,B.H and Altae, A, (1994),"Stress and settlement of footings in sand " Proceedings of ASCE, Geotechnical Special Publication, No.40 College Station, Texas.
[8] Horvart, Stolle (1996) Frustum confining vessel for testing model piles, Canadian Geotechnical Journal. 33; 499-504.
[9] Meymand, Shaking table scaled model tests of non-linear soil-pile superstructure interaction in soft clay, 1998 ,Berkeley, PhD thesis.
[10] Ozkahriman, F (2011), " Physical and Numerical dynamic response modeling on slopes and embankments ", Purdue University,PhD Thesis.
[11] Roscoe, K.H., 1968, "Soils and Model testJournal of Strain Analysis, Vol.3 No.1, pp.57-64.
[12] Roscoe, K.H. and Poorooshasb, H., 1963.A Fundamental Principle of Similarity in model Test for earth pressure problems. Proceedings of the 2 nd Asian Regional Conferrence on Soil Mechanics, Bangkok.
[13] Sedran, G et.al, 2001, "An Investigation of Scaling and dimensional analysis of axially loaded piles", Canadian Geotechnical Journal.
[14] Sulaeman,A (2010), " The use of lightweight concrete pile for foundations on soft soils", PhD thesis.
[15] Wan Nor Hanisah Wan Hamzah, Sulaeman,A (2010) , " Critical state parameters on clay and residual soil " UTHM, BSc thesis.
[16] Wood, D.M. (2004), " Geotechnical Modelling ", Spon Press (USA).
[17] Wood, D.M, Adam, C and Taylor,C (2012), Shaking table testing of geotechnical " International Journal of Physical Modelling in Geotechnics (IJPMJ).
[18] Zelikson, A (1996), Geotechnical model using the hydraulic gradient method. Geotechnique, 19; 495-508.